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Prestressed Wood Bridges

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EXECUTIVE SUMMARY

Transverse prestressing of wood was conceived in 1976 as a method for rehabilitating nailed laminated wood decks. Since then the concept has become the subject of a major development program, aimed at revitalizing wood as a viable design alternative in bridge construction. This paper highlights the development of this type of construction from its original application to existing bridges, to the subsequent research and development leading to its inclusion in the Ontario Highway Bridge Design Code. Various field applications are discussed, including the recent design and construction of the first prototype prestressed wood bridge. Transverse prestressing increases load distribution and results in a very efficient load sharing system. It promotes the use of lower grade materials, utilizes renewable resources and requires far less energy than other construction materials.

1/ INTRODUCTION

In the past, vertically laminated wood decks were constructed by successively nailing each laminate to the previous one to form a continuous wood deck. A typical installation is the Hebert Creek Bridge, a cross-section of which is

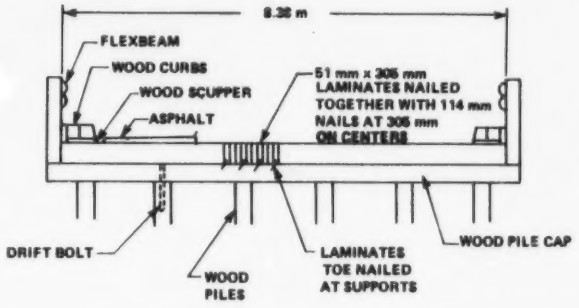


Fig. 1 Hebert Creek Bridge, Cross-Section.

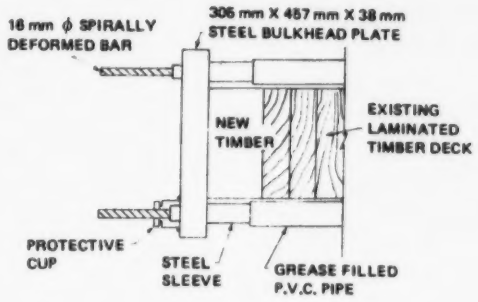
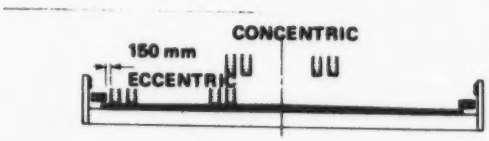

16 mm ϕ SPIRALLY DEFORMED BAR
305 mm X 457 mm X 38 mm STEEL BULKHEAD PLATE
NEW TIMBER
EXISTING LAMINATED TIMBER DECK
PROTECTIVE CUP
STEEL SLEEVE
GREASE FILLED P.V.C. PIPE

Fig. 2 Hebert Creek Bridge, Prestressing Detail.

shown in Figure 1. The load carrying capacity of such bridges is directly dependent upon their ability to distribute wheel loads among the laminates, which is a function of the holding power of the nails. Due to the repetitive action of heavy commercial vehicles, this holding power diminishes after some years of service. The nails eventually crush the wood and deform, allowing the laminates to separate and move independently. This delamination reduces the capacity of the bridge, and accelerates breakup of the wearing surface, thus allowing access of deleterious materials.



The Hebert Creek bridge displayed the characteristic delamination associated with these decks, and was selected as a test site for the development of a prestressing system [1], [2]. This system would apply pressure perpendicular to the laminates, closing all spaces, and creating adequate friction between the wood surfaces to re-establish load distribution. The prestressing system used at Hebert Creek consists of high strength bars located above and below the deck. These bars are attached to heavy steel plates to form a strong, flexible clamping system as shown in Figure 2. Figure 3 indicates that prestressing reduced the vertical displacements of the deck by 50 percent. The excellent performance of the Hebert Creek bridge has stimulated a major research and development program [3], [4], including the extension of this concept to new structures.

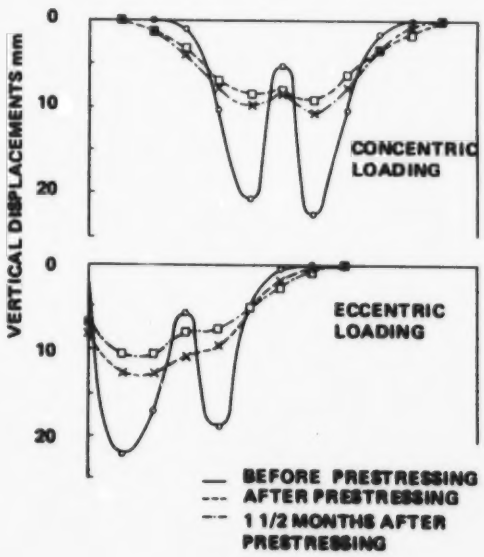


Fig.3 Hebert Creek Bridge, Midspan Deflections.

The successful prestressing of the Hebert Creek Bridge demonstrated the benefits which could be realized by transversely prestressing wood decks and stimulated an extensive program of research and development. The work has been reported elsewhere [3],[4],[5],[6],[7], and only the more important findings are given here.

2.1 Factors Affecting Prestress Level

If transverse prestressing is to be of lasting benefit it is essential that an acceptable level of prestress be maintained throughout the life of the structure. Thus a thorough understanding of the loss of prestress with time and of the factors affecting that loss is required.

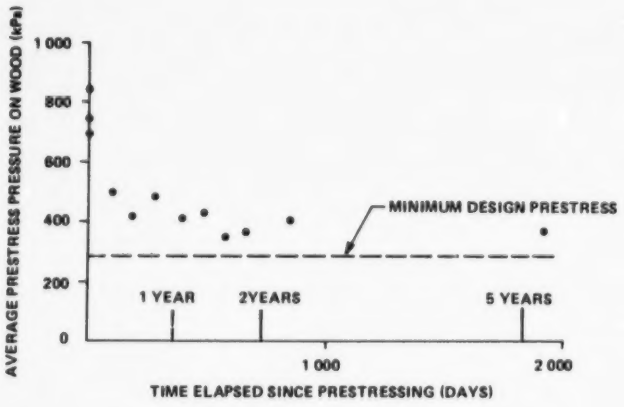


Fig. 4 Hebert Creek, Prestress Level

The Ontario Ministry of Transportation and Communications (MTC) conducted a pilot study of the behaviour of 38 mm x 240 mm wood laminates compressed perpendicular to the grain [1] prior to prestressing the Hebert Creek bridge. This study suggested that a residual level of prestress could be maintained for considerable time after stressing and that this level was greatly dependent upon the stiffness of the prestressing system. As a partial check of these results, the prestress in the Hebert Creek bridge has been measured regularly. Although there are short term fluctuations of prestress as shown in Figure 4, it appears that the mean prestress level is approaching a virtually stable residual value. It has not been necessary to restress this bridge since its rehabilitation in 1976.

Queen's University at Kingston was requested by MTC to conduct an extensive study of factors affecting prestress level in a laminated wood system [5], [7]. The effects of initial level of prestress, stiffness of the prestressing system, frequency of restressing, relative humidity, temperature and preservative treatment were studied for specimens of Ontario white pine, Ontario red pine and western hem-fir.

2.1.1 Test Program

The transverse prestressing of a wood deck was simulated in test specimens consisting of thirteen 50 mm thick, 250 mm square laminates placed in a test rig designed to introduce compression perpendicular to the grain. The specimens were placed between two steel plates which were interconnected by four high strength steel threaded rods as shown in Figure 5, or by a central prestressing strand, depending on the axial

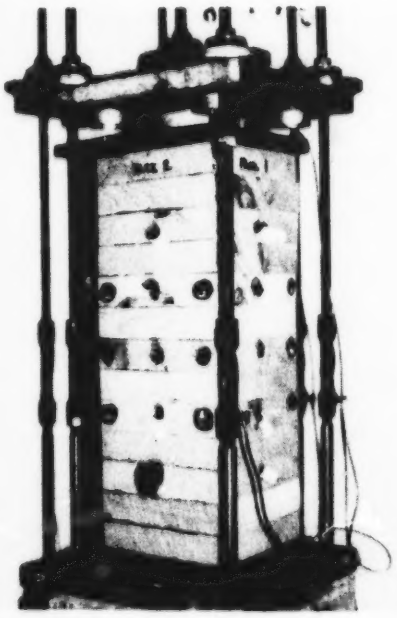


Fig. 5 View of Relaxation Rig.

stiffness desired for the particular rig. Prestress was applied by tightening the nuts on the threaded rods, or by standard prestressing techniques for the rigs with the prestressing strand. The compression forces in the wood were continuously monitored by compression load cells.

Three rooms with controlled temperature and relative humidity were established to accommodate a total of sixteen test rigs. The time dependent behaviour of the specimens was predominantly one of loss of stress under constant deformation and the term "relaxation" was adopted to describe their behaviour.

2.1.2 Typical Relaxation

The results of the relaxation tests are best expressed in terms of relaxation curves. In order to simplify presentation the term "stress ratio" is introduced, and defined as the ratio of the stress at anytime, to the initial prestress. This ratio, expressed as a percentage, is plotted against the time elapsed since the application of prestress.

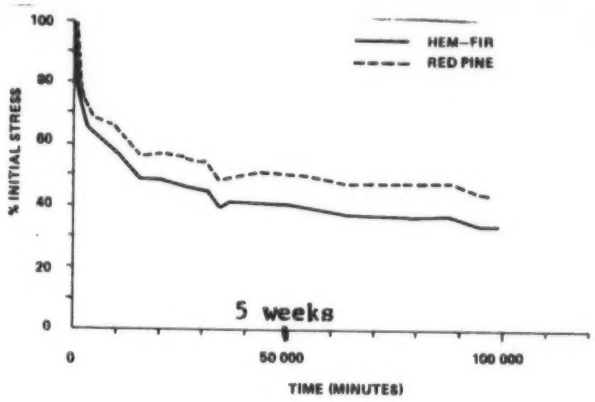


Fig. 6 Typical Relaxation after Second Stressing

Figure 6 shows typical relaxation curves obtained after second stressing of the specimens to 700 kPa in the stiffer rigs. In each case the first stressing of these specimens consisted of the application of a prestress of 700 kPa and subsequent relaxation for about 5 weeks. The temperature and relative humidity were kept sensibly constant at 18°C and 60% throughout the period of relaxation. It can be seen that the stress ratio falls rapidly immediately after stressing, but becomes more or less asymptotic to an equilibrium value after approximately 50,000 minutes. It was determined from sixteen tests that under these conditions the stress ratio 5 weeks after second stressing appeared to be independent of the species group and varied from 36% to 68%. The effect of reducing rig stiffness in order to give a more realistic representation of field conditions, is discussed in Section 2.1.4.

2.1.3 Prestress Level

The relaxation tests, in which the initial prestress varied from 550 to 1000 kPa, showed that relaxation behaviour was insensitive to the value of initial prestress within this range. As a result an initial prestress level of 700 kPa is recommended as the optimum for the three species groups in this study, in order to avoid crushing while at the same time eliminating most of the warp in the laminates.

2.1.4 Stiffness Ratio

The stiffness ratio is defined as the ratio of the axial stiffness of the stressing system to that of the wood being stressed. The effect of varying stiffness ratio was examined using the two types of rigs described in Section 2.1.1.

Figure 7 shows the relaxation after second stressing of nominally identical specimens of creosoted hem-fir under constant temperature and varying relative humidity in rigs of two different stiffnesses. The stress ratio for the wood in the stiffer rig 50,000 minutes after stressing is 50%. This lies within the range of values for typical relaxation discussed in Section 2.1.2 and provides some evidence that the relaxation characteristics of wood are not affected by the creosote treatment. The comparable stress ratio of 74% for the wood in the rig with the lower stiffness is evidence of the more desirable relaxation behaviour which can be expected as the stiffness ratio is decreased. The lower stiffness ratio is representative of existing field structures which have stiffness ratios in the range of 1.1 to 1.3. The results of this investigation enabled the definition of practical upper limits of stiffness ratio for prestressed systems.

2.1.5 Relative Humidity

The most important environmental condition affecting the stress ratio was found to be the relative humidity. Figure 7 shows the effect on stress ratio of a change in the relative humidity from 60% to 80% for a period of 70,000 minutes (7 weeks). The increase in relative humidity is accompanied by an increase in volume of the wood and a corresponding increase in stress ratio. The wood begins to respond to the change in humidity almost instantaneously but the time taken for the wood to reach equilibrium with the new conditions is approximately the same as that required after initial stressing. A study of weather office records showed that although the relative humidity can change at a rate comparable to that in these tests, the humidity is not maintained at the new level for the same length of time. Thus, in practical applications the response of the wood would be attenuated and the effect of change in relative humidity depicted in Figure 7 may be considered to be an upper bound to the response under site conditions.

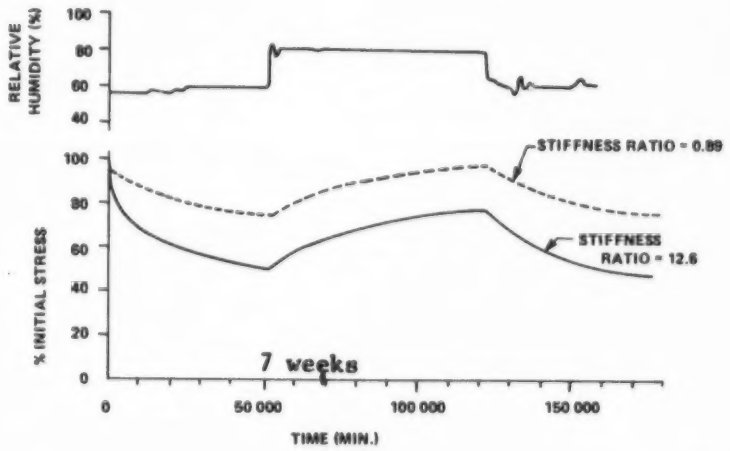


Fig. 7 Relaxation, Variable Humidity and Rig Stiffness

Figure 7 also shows that when the relative humidity is restored to its original level, the stress ratio eventually reaches an equilibrium value equal to that which might have been expected in the absence of any disturbance. That is, provided the initial stress is not exceeded during the period of increased humidity there is no cumulative loss of prestress attributable to the humidity change.

2.1.6 Frequency of Stressing

It will be noted that Figures 6 and 7 present the relaxation of the specimens after second stressing. It was found in this study that the loss of prestress after first stressing was large, and appeared to be associated with removing or reducing warp in the wood. Figure 4 shows the variation of prestress in the Hebert Creek bridge which has been subjected to only one stressing. In this case, considerable time (2 weeks) was required to prestress the bridge due to

limited availability of jacking equipment. It is believed that this slow application of prestress contributed to removal of warp.

If prestressing of a structure is completed within a short period of time, it is believed that some restressing will be necessary and relaxation tests were conducted with the object of establishing an optimum frequency of stressing.

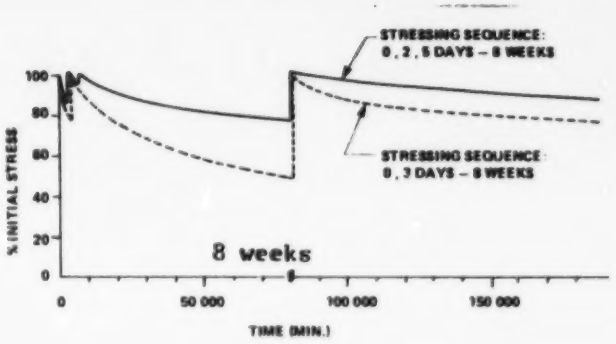


Fig. 8 Relaxation, Repeated Prestressing.

Eight possible stressing sequences were studied. The results showed that the long term residual stress ratio could be substantially increased with the introduction of additional stressings at appropriate intervals. The results further indicated that to achieve maximum long term benefit, re-stressing should not be performed until relaxation after the previous stressing was virtually complete. This conflicts with practical considerations which dictate that the re-stressing should be completed within the shortest time possible.

Figure 8 is a record of the stress ratio in a specimen of creosoted hem-fir, from the initial stressing, through a number of restressings for one of the more promising and one of the least promising stressing sequences. Rigs with the lower stiffness were used in these tests; temperature and relative humidity were kept constant at 18°C and 60% respectively. The figure shows that the stress ratio does not at any time fall below 50% for the least desirable of the two stressing sequences, nor below 75% for the recommended sequence. There is, therefore, a real advantage in introducing an additional stressing, preferably within about a week of the first stressing.

The specimens from which the results in Figure 8 were obtained continue to be monitored and at 24 weeks after the last stressing, the stress ratio in the specimens subjected to the 0,2,5 day, 8 week sequence is virtually constant at a level of 82%.

2.2 Orthotropic Plate Parameters

The analysis and load testing of the Hebert Creek bridge (1) indicated that orthotropic plate theory may be applicable to prestressed wood decks. However, this theory requires the determination of several important elastic parameters, in particular, the flexural and torsional stiffnesses of the deck. Queen's University was requested to conduct studies to confirm the applicability of this analysis technique, and to establish any necessary elastic parameters. The main variables in this study were wood species and level of prestress which was varied over a range of 275 kPa to 1000 kPa depending on the material.

2.2.1 Idealization of decks

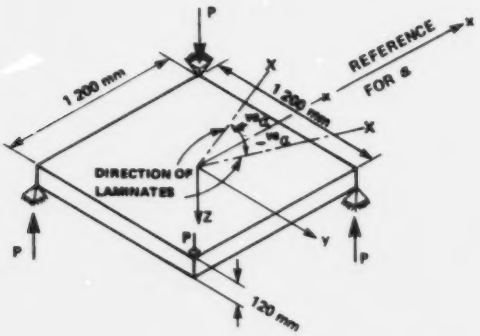
The test arrangement shown in Figure 9a was used in the determination of the orthotropic plate parameters and is based upon a method first proposed by Tsai [8]. In the general orthotropic plate of Figure 9(a), x,y,z are the reference coordinate axes, and X and Y represent the elastic symmetry axes. The assumed principal directions of orthotropy, L,T,R, of a typical laminate are shown in Figure 9(b).

The typical laminated wood deck, in which the laminates are parallel (or perpendicular) to the x or y directions (i.e. α=0), is termed "specially orthotropic". The governing equation for an element in such a system subjected to loading $p(x,y)$ is:

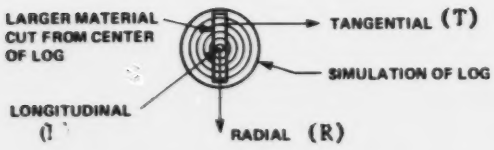
$$D_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + D_y \frac{\partial^4 w}{\partial y^4} = p(x,y) \qquad (1)$$

The flexural rigidities, D_x,D_y, and the effective torsional rigidity, 2H, are related to the elastic moduli, E_x,E_y, the torsional rigidity D_{xy}, the shear modulus, G_{xy}, and the appropriate Poisson's ratios.

In the case of a deck having principal axes of material orthotropy (L,T) coinciding with the x-y coordinate system, the subscripts x and y in Eqn. (1) are interchangeable with L and T, respectively. The plate parameters can then be defined in terms of the material parameters, E_L, E_T, G_{LT} and the corresponding values of Poisson's ratios as shown in Table 1.



a) ARRANGEMENT FOR PLATE TWIST TEST



b) ASSUMED PRINCIPAL DIRECTION OF ORTHOTROPY OF LAMINATES

Fig. 9 Idealization of Orthotropic Plate.

In Tsai's method [8] a square plate is subjected to uniform twisting and bending moments by supporting three corners as shown in Figure 9(a) and applying a point load, P, at the fourth corner. Displacements w_o and w, at the centre and loaded corner respectively, are measured for varying load and the slopes of the load-deflection plots, P/w and P/w_o are used in the evaluation of plate parameters. In order to determine the elastic constants completely, it is theoretically necessary to test a series of square plates constructed such that $\alpha = 0^o$, $+45^o$, and -45^o respectively. (Fig.9(a)). In addition, tests on appropriately laminated beam specimens should be conducted to determine E_L and E_T, the apparent moduli of elasticity in the principal directions of orthotropy.

2.2.2 Laboratory Tests

The typical dimensions of the plate specimens are shown in Figure 10.

Table 1. Expressions for Orthotropic Plate Parameters

Quantity	Expression	Quantity	Expression
θ_1	$1/(1-\mu_{LT}\mu_{TL})$	D_x	$E_{LT} \frac{t^3}{12} \theta_1$
θ_2	$\mu_{TL}/(1-\mu_{LT}\mu_{TL})$	D_y	$E_T \frac{t^3}{12} \theta_1$
θ_3	$\mu_{LT}/(1-\mu_{LT}\mu_{TL})$	D_{xy}	$G_{LT} \frac{t^3}{12}$
H	$D + 2D_{LT} = E_L \frac{t^3}{12} \theta_2 = E_T \frac{t^3}{12} \theta_3$	D	$E_L \frac{t^3}{12} \theta_2 = E_T \frac{t^3}{12} \theta_3$

Prestress was applied by means of bars, or wires, located at the middle surface. In the ± 45° plates it was necessary, as shown in Figure 10, to have some discontinuous projections outside the area defined by the square plate. However, in the early studies [5] it was shown that these projections had little or no effect on plate deflections. The +45° plate was obtained from the -45° plate by rotating the latter with respect to the supports.

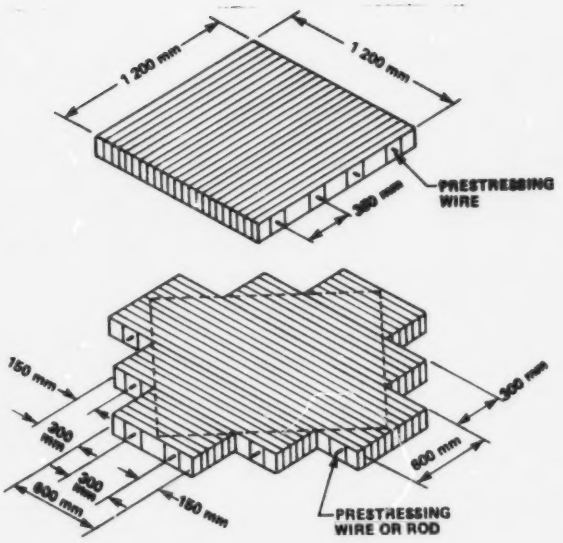


Fig.10 Typical Plate Specimens

Initially, the value of E_L was determined from loading tests on simply supported prestressed, longitudinally laminated beams, consisting of five laminates each being about 120 mm deep by 1830 mm long. However, it was found the E_L determined from these tests did not vary with prestress and compared very favourably with an average value determined from tests on a large number of individual laminates. Therefore, the need to test a large number of prestressed laminated beams was eliminated. Values of E_L determined from tests on thirty beams of each material are summarized in Table 2.

Table 2 Summary of E_L Values Obtained from Beam Tests

Material	E_L (MPa)	Std.Deviation (MPa)	Coeff. of Variation
Hem-Fir	11,510	2,285	0.20
Red Pine	7,710	2,205	0.28
White Pine	7,500	1,220	0.16

The tests were performed on rough-sawn, untreated specimens at 10-12 percent moisture content. Midspan loading was used with a simply supported span of 1100 mm. The values of E_L were multiplied by a factor 1.18 to account for shear deflection. Initial testing of transversely laminated beams proved to be unfeasible [5] and, therefore, the values of E_T were determined from the ±45° plate tests.

The plate tests indicated that the parameters G_{LT} and E_T varied linearly with prestress, and generally upon reloading had values higher than those obtained during first loading, apparently due to "set" in the system. As expected, the results exhibited some scatter; however, it was possible to develop lower bound values for all the species combined. As an example, Figure 11 shows the variation of G_{LT} with prestress. Variation of E_T was similar to that of G_{LT}.

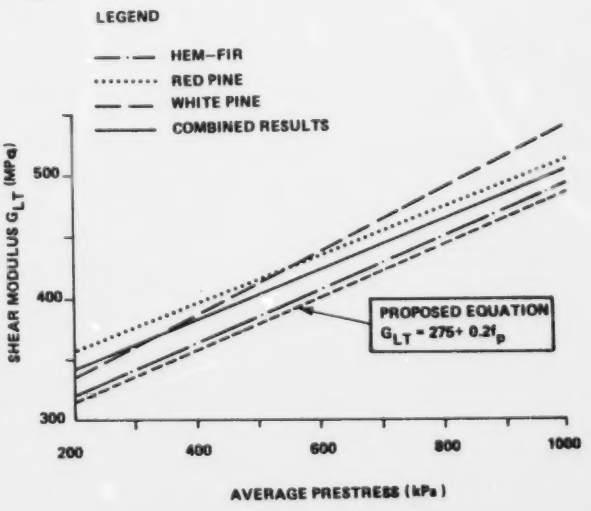


Fig. 11 Shear Modulus vs. average Prestress

The following lower bound values are recommended for design purposes:

$$G_{LT} = 275 + 0.2\ f_p \qquad E_T = 100 + 0.2\ f_p$$

where G_{LT} and E_T are in units of MPa and the average prestress f_p is in kPa. Values obtained for the modification factors θ_1,θ_2, and θ_3 were 1.02, 0.02 and 0.55, respectively. It was determined [6] that in the future the -45° plate tests could be eliminated, thereby expediting testing considerably. The above values of the plate parameters were used in a finite element analysis of a representative rectangular plate. The theoretical results compared favourably with the experimental results of the plate [5], [6]. It is the intention of MTC to conduct a number of field tests to generate results for checking the reliability of the recommended values of the parameters.

2.3 Interlaminate Shear Strength

The distribution of concentrated loads on a laminated deck is dependent on the interlaminate shear strength, τ_μ, between the laminates. The ultimate shear strength is likely to be dependent on the prestress level, species of wood and the surface conditions of adjacent laminates. A number of "push-out" tests, using a rig shown in Figure 12 were conducted to determine representative values of τ_μ. Five laminates, approximately 250 mm square by 50 mm thick, were placed side-by-side between two steel plate bulkheads as shown in Figure 12. The required uniform normal pressure was applied by tightening the four threaded bars. A central force was then applied to the middle three laminates, and the level of loading was increased until slip occurred. Several tests were conducted on a particular specimen, with the slipped surfaces returned to their initial positions and the prestressed level restored.

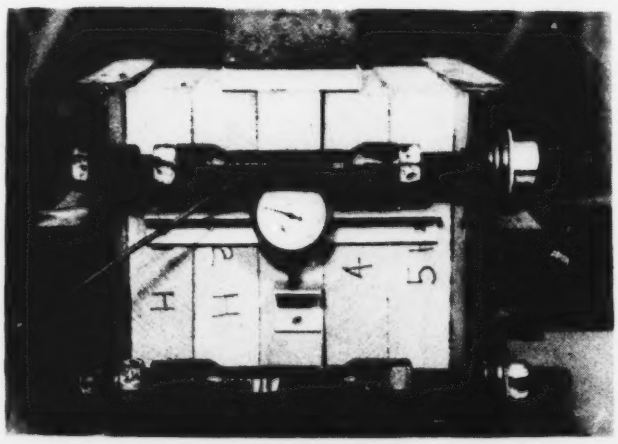


Fig.12 View of Friction Rig.

In some specimens, the surfaces were left in the rough sawn condition, while in others the surfaces were sanded. A third condition was introduced by using some hem-fir specimens treated with oil-borne creosote. Figure 13 shows plots of τ_μ against prestress, f_p, indicating different linear relationships depending on surface conditions.

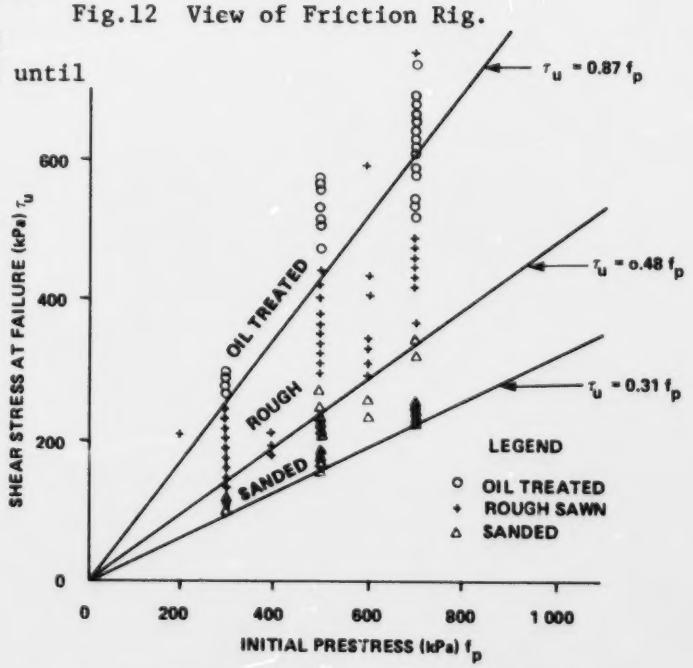


Fig. 13 Shear Strength vs. Initial Prestress

2.4 Load Sharing

One of the most important

aspects of the prestressed wood system is the significant effects of load sharing at the ultimate limit state. If strength tests are performed on a sufficiently large number of wood specimens, the assumed distribution of strength of the individual members is believed to be described by a normal or log-normal function. The design strength for the population is usually based on the 5th percentile of the distribution. This approach is taken, in order to ensure that the weaker material defines the strength of the structural system. However, in a load sharing system, particularly one which contains a large number of individual elements, the weaker material will not necessarily define the overall capacity at the ultimate limit state.

Table 3. Summary of In-grade Test Results

TEST MODE	NUMBER TESTED	STRENGTHS MPa	WHITE PINE	RED PINE	WESTERN HEMLOCK
INDIVIDUAL SPECIMENS	60	MEAN STRENGTH 5TH. PERCENTILE	15.98 8.75	27.98 14.87	45.92 25.90
DECK OF 6 LAMINATES	10	MEAN STRENGTH 5TH. PERCENTILE	19.40 15.02	27.52 18.99	46.89 35.63
DECK OF 12 LAMINATES	10	MEAN STRENGTH 5TH. PERCENTILE	21.17 19.00	29.02 22.76	43.89 39.56
DECK OF 18 LAMINATES	10	MEAN STRENGTH 5TH. PERCENTILE	18.31 15.97	28.78 25.99	44.83 39.07
% INCREASE AT 5TH. PERCENTILE INDIVIDUAL TO 18 LAMINATE DECKS			82.5	74.9	50.8

To develop a data base for determining the design strength of prestressed wood decks. MTC initiated an extensive ingrade testing program which involved the use of full size graded material. The Western Forest Products Laboratory (WFPL) in British Columbia, was contracted to conduct the ingrade testing in both individual and load sharing modes. The work is fully documented in Reference [9]. Three species of kiln dried material were tested, namely western hemlock, Ontario red pine and Ontario white pine. All the material was 51 mm x 254 mm, rough sawn, and approximately 4.9 m in length. About 60 pieces from each group were tested as individual members, and the rest were used to construct prestressed decks comprised of 6, 12 and 18 laminates. The results of the tests are summarized in Table 3, which gives the mean and the 5th percentile strengths as related to log-normal distributions. Although the mean strengths were not affected appreciably by introducing the specimens into a load sharing system, the 5th percentile strengths increased from 51 to 83 percent.

It is possible to demonstrate the effects of load sharing in laminated decks, by performing a statistical evaluation [4] using the results of the tests from the WFPL. Initially, the results of the tests on the individual specimens were plotted, and the strength-stiffness relationships as shown in Figure 14, were established. Subsequently, the test results were related to normal functions, similar to that shown in Figure 15. The coefficients of variation of strengths derived from the tests were about 35 percent for red pine and white pine, and 32 percent for western hemlock. If this normal function is assumed to describe the distribution of strength of the individual

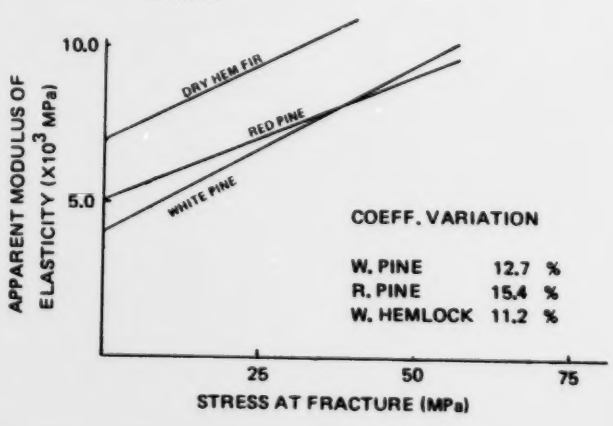


Fig. 14 Apparent Modulus vs. Strength

laminates in a deck, the following argument may be advanced. The area under the curve can be divided into N equal areas as shown in Figure 15. The dividing line between the j^{th} and $(j+1)^{th}$ sub-area will represent the rupture strength of the j^{th} laminate. By combining this with the strength-stiffness relationships in Figure 14, a laminated deck can be simulated in which the failure of each laminate can be predicted. The process involves the use of exponential functions and does not lend itself to closed format manipulation. However the total moment in a deck can be determined as each laminate reaches its rupture strength. The results of this procedure are shown in Figure 16 for a simulated 24 laminate deck comprised of red pine. The peak moment occurs after at least 6 laminates have failed. This applies also to the other two species of wood. The WFPL tests confirmed that several fractures can be expected to occur before the maximum strength of the prestressed deck is reached. A comparison in Table 4 of the peak moments with those derived using the 5th percentile strengths demonstrates the increase in capacity that can be realized by using a variable strength material in a load sharing system.

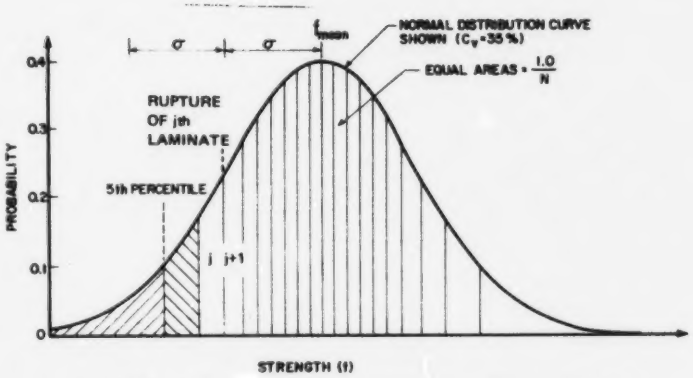


Fig. 15 Normal Distribution Function

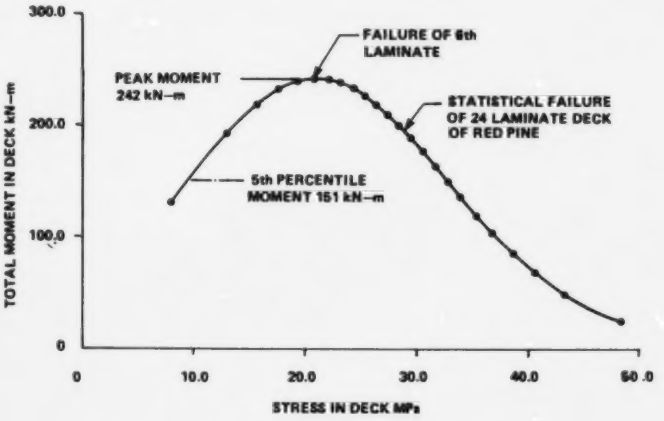

FAILURE OF 8th LAMINATE
PEAK MOMENT 242 kN-m
STATISTICAL FAILURE OF 24 LAMINATE DECK OF RED PINE
5th PERCENTILE MOMENT 151 kN-m
TOTAL MOMENT IN DECK kN-m
STRESS IN DECK MPa

Fig.16 Failure of Simulated Deck

This approach was used in conjunction with the results of the in-grade tests performed at the WFPL, to derive modification factors given in the Ontario Highway Bridge Design Code (OHBDC) [br. for load sharing in prestressed wood deck decks.

Table 4. Summary of Strengths of Simulated Decks.

SPECIES	5th PERCENTILE (kN-m)	PEAK MOMENT	INCREASE
White Pine	90	142	58%
Red Pine	151	242	60%
W. Hemlock	282	405	44%

2.5 Cyclic Testing

A preliminary study was performed at Queen's University to evaluate the effects of cyclic loads on transversely prestressed wood decks, as compared to the conventional nailed system. Two 6 m simply supported beams were constructed, each comprised of eight longitudinal wood laminates. Discontinuities or butt joints were introduced according to the MTC standard details for continuous laminated

decks, which resulted in only two of the eight laminates being continuous from support to support in each beam. One of the two beams was assembled according to conventional nailing requirements and the other was transversely prestressed to approximately 345 kPa. Each of the laminated beams was to be subjected to 500,000 cyles of loading producing peak stress equivalent to that at service load levels.

The testing of the conventional nailed deck was terminated at about 95000 cycles, since the deflections were becoming increasingly high and evidence of severe delamination was present. Shear failure of the nails in the vicinity of the butt joints was recognized as early as 64000 cycles and movement at the butt joints noticeably impaired the integrity of the deck. The nail connection in this system leads to high stress concentrations and the premature failure at the connection under repeated loads. The transversely prestressed beam sustained the required 500,000 cycles of load with only minor evidence of butt joint movement.

The preliminary cyclic tests provided evidence of the superior ability of the transversely prestressed decks to withstand the effects of repeated loads. The MTC is presently formulating an investigation which will extend this work to include cyclic testing of wider decks subjected to concentrated loads.

3. IMPLEMENTATION

The transverse prestressing of wood has been incorporated into a number of field applications involving both rehabilitation and new construction.

3.1 Rehabilitation

Figure 2 illustrates the prestressing system originally used for the Hebert Creek bridge. The prestressing bars were enclosed in grease-filled PVC tubes for protection. To accommodate movement during prestressing, the tubes were designed to slide over steel sleeves welded to the anchorage plates. This system led to several operational problems. Rotation of the anchorage plates caused crushing of the wood and inhibited movement of the PVC tubing over the steel sleeves.

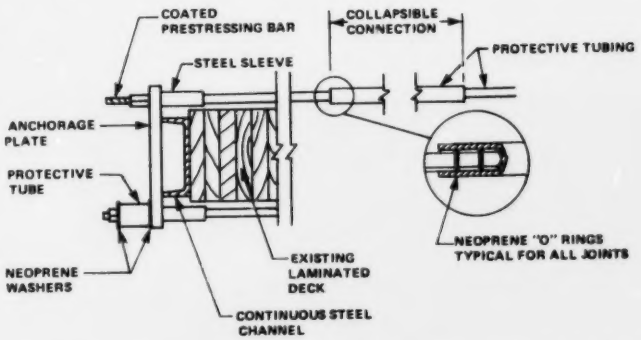


Fig.17 Prestressing System for Rehabilitation

Fig.18 Pickeral R. Bridge Rehabilitation

Since the successful upgrading of the Hebert Creek bridge, five other longitudinally laminated wood decks have been successfully rehabilited by prestressing. During the course of this work numerous refinements were developed, leading to the system detailed in Figure 17. To prevent individual rotation of the anchorage plates and increase distribution of the prestress forces, a continuous steel channel bulkhead has been adopted. Since some overall rotation of the bulkheads cannot be avoided, the collapsible connection for the protect-

ive tubing is now located away from the edge of the deck. The protection of the bars has also been improved; the bars are either galvanized or epoxy-coated and all overlapping joints of the protective tubing are provided with neoprene "O" rings, to prevent access of deleterious materials.

Various installation techniques have been used to rehabilitate existing decks. It was found that the most efficient method was the complete removal of the existing wearing surface as shown in Figure 18. To expedite the prestressing operation MTC designed a 24 jack hydraulic system.

There are hundreds of nailed laminated decks in Ontario. It is believed that all of these decks will eventually require rehabilitation or replacement. A report which will review the work done to date and provide recommended design details is currently being prepared.

3.2 New Construction

The prestressing system described for rehabilitation has been modified for use in new installations. New prestressed decks will incorporate a single bar system as shown in Figure 19. The bars are provided with better protection by passing them through pre-drilled holes at mid-depth of the laminates. Also, placement of the asphalt wearing surface is facilitated, since the bars no longer obstruct the top surface of the deck.

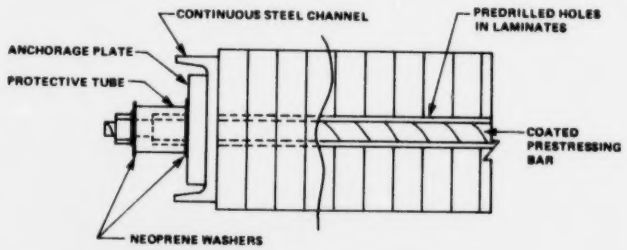


Fig.19 Prestressing System for New Construction.

To evaluate the effectiveness of this new design concept, MTC, in cooperation with the Ministry of Natural Resources (MNR), has designed and supervised the construction of a prototype structure on an MNR access road near Espanola, Ontario. The structure, shown in Figure 20, was designed as a frame with inclined legs in order to take advantage of the site conditions. Prestressed laminated wood was used in the deck and legs which were pre-assembled, using galvanized nail plates. The individual laminate frames shown in Figure 21 were lifted into place using a crane. The frames were placed side by side, the prestressing bars were then passed through the predrilled holes and attached to the steel anchorages along the sides of the bridge. The structure is performing satisfactorily and will be load tested in 1982.

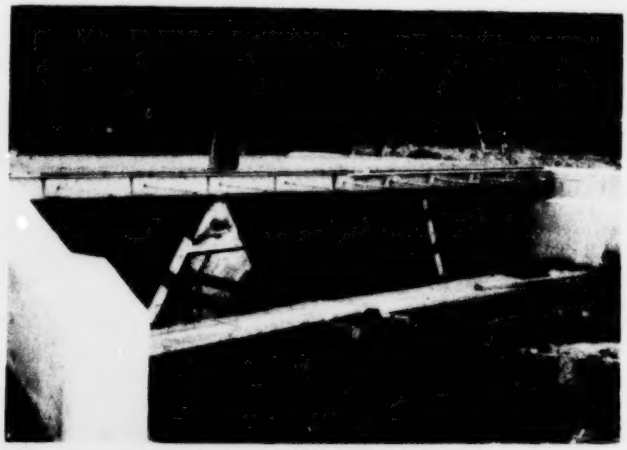


Fig.20 Fox Lake Bridge, MNR Sudbury.

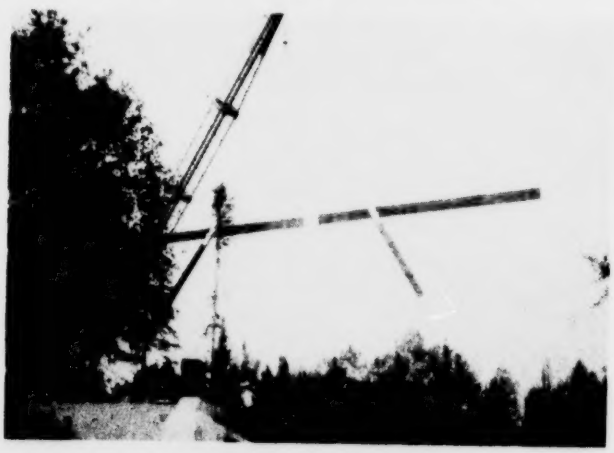


Fig.21 Fox Lake Bridge Assembly

Another new prestressed wood deck is being constructed as part of the rehabilitation of a major wood

bridge in north western Ontario. The Sioux Narrows bridge, shown in Figure 22, is believed to be the longest single span wood bridge in North America. It was built in 1936 of B.C. Fir and spans 64 m over the Lake-of-the-Woods near Kenora. The bridge contains a deck system which is identical to that of many smaller wood truss bridges built at that time. As illustrated in Figure 23 the existing deck system consists of a transverse laminated nailed wood deck, on longitudinal wood stringers. The stringers are supported by trussed wood floor beams, which rest on the bottom chords of the main trusses. A smaller wood truss bridge near North Bay, which has a deck system identical to that of the Sioux Narrows bridge was load tested. The results indicated that these deck systems do not provide adequate longitudinal load distribution. As a result, the steel bars trussing the floor beams become over-stressed.

The rehabilitation of the Sioux Narrows bridge requires, among other things, the replacement of the transverse laminated deck and a method for increasing the longitudinal stiffness of the deck system. Both of these requirements were accommodated by introducing a longitudinally laminated, prestressed deck, as illustrated in Figure 23. It was verified by analysis that the reduction in transverse stiffness, introduced by reorienting the laminated deck longitudinally, was not detrimental to the capacity of the structure. However this reorientation increases the longitudinal stiffness, and hence, distributes the live load more effectively thus reducing the stresses in the floor beams. Further, since prestressing inhibits relative movement between laminates, the rehabilitated structure can support a continuous asphalt wearing surface.

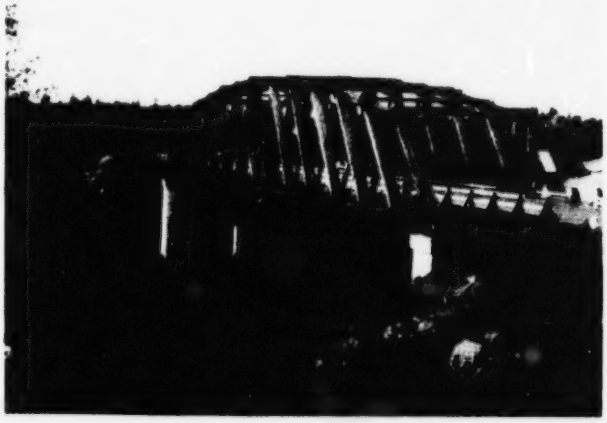


Fig.22 Sioux Narrows Bridge , Kenora.

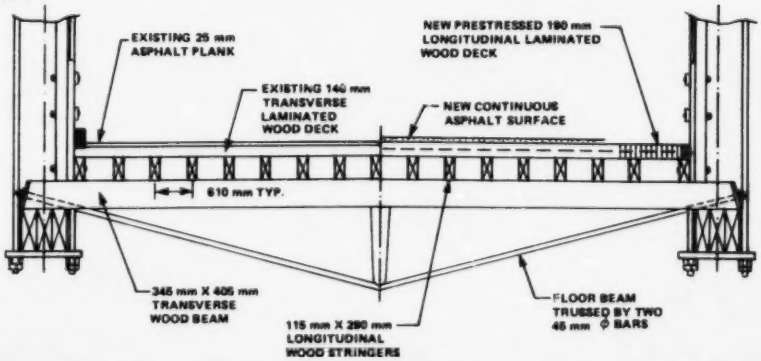

EXISTING 25 mm ASPHALT PLANK
NEW PRESTRESSED 190 mm LONGITUDINAL LAMINATED WOOD DECK
EXISTING 140 mm TRANSVERSE LAMINATED WOOD DECK
NEW CONTINUOUS ASPHALT SURFACE
610 mm TYP.
345 mm X 405 mm TRANSVERSE WOOD BEAM
115 mm X 290 mm LONGITUDINAL WOOD STRINGERS
FLOOR BEAM TRUSSED BY TWO 45 mm ϕ BARS

Fig. 23 Sioux Narrows Bridge, Deck Cross Section.

4.0 CONCLUSIONS

The extensive research and development, outlined in this paper, together with the experience gained during field applications and load testing, have enabled the formulation of design specifications for prestressed laminated wood decks. The draft specifications are intended for adoption in the 1982 edition of the OHBDC [10] and enable the extension of the prestressed wood concept to new structures.

The greatly enhanced load distribution as observed in the initial field application at the Hebert Creek bridge, is one of the main benefits of transversely prestressing wood. Values for the orthotropic plate parameters and interlaminate shear strengths have been established, and included in the OHBDC specifications.

The implications of maintaining a minimum level of prestress in a laminated wood deck has been examined. Monitoring of new and rehabilitated structures supports

the findings of the laboratory studies, relating to the time-dependent behaviour of the prestress level in these decks. The studies have led to recommendations for levels of initial prestress, maximum stiffness of prestressing system and guidelines for the number and frequency of restressings.

The OHBDC is based upon limit states design principles which is a relatively new concept in the design of wood structures. The Code recognizes the fact that benefits can be derived from load sharing at ultimate limit states and the draft specifications include modification factors for load sharing in prestressed wood decks.

In addition to design parameters the draft specifications include a design commentary and various system details for both new and existing structures.

Wood is a renewable resource, requiring far less energy to produce than either cement or steel. It is readily available in Ontario, in the sizes and grades required for this type of construction and is a material which can be handled without skilled labour or sophisticated equipment. With the development of this new concept in wood design, which produces stronger and more durable structures, there is little doubt that wood will find increasing favour in the years ahead.

5.0 ACKNOWLEDGEMENTS

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- North Bay District Bridge Crew.

Ontario Ministry of Natural Resources

- H. Walsh, Regional Engineer, Sudbury, Ontario.
- S. Cross and Huntsville Field Construction Crew.

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